Chapter 5 Contents

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Chapter 5 Engineering Properties of Soil and Rock

5.1 Overview

The purpose of this chapter is to identify, either by reference or explicitly herein, appropriate methods of soil and rock property assessment, and how to use that soil and rock property data to establish the final soil and rock parameters to be used for geotechnical design. The final properties to be used for design should be based on the results from the field investigation, the field testing, and the laboratory testing, used separately or in combination. Site performance data should also be used if available to help determine the final geotechnical properties for design. The geotechnical designer's responsibility is to determine which parameters are critical to the design of the project and then determine those parameters to an acceptable level of accuracy. See **GDM Chapter 2**, and the individual chapters that cover each geotechnical design subject area, for further information on what information to obtain and how to plan for obtaining that information.

The focus of geotechnical design property assessment and final selection shall be on the individual geologic strata identified at the project site. A geologic stratum is characterized as having the same geologic depositional history and stress history, and generally has similarities throughout the stratum in terms of density, source material, stress history, and hydrogeology. It should be recognized that the properties of a given geologic stratum at a project site are likely to vary significantly from point to point within the stratum. In some cases, a measured property value may be closer in magnitude to the measured property value in an adjacent geologic stratum than to the measured properties at another point within the same stratum. However, soil and rock properties for design should not be averaged across multiple strata. It should also be recognized that some properties (e.g., undrained shear strength in normally consolidated clays) may vary as a predictable function of a stratum dimension (e.g., depth below the top of the stratum). Where the property within the stratum varies in this manner, the design parameters should be developed taking this variation into account, which may result in multiple values of the property within the stratum as a function of a stratum dimension such as depth.

5.2 Influence of Existing and Future Conditions on Soil and Rock Properties

Many soil properties used for design are not intrinsic to the soil type, but vary depending on conditions. In-situ stresses, the presence of water, rate and direction of loading can all affect the behavior of soils. Prior to evaluating the properties of a given soil, it is important to determine the existing conditions as well as how conditions may change over the life of the project. Future construction such as new embankments may place new surcharge loads on the soil profile or the groundwater table could be raised or lowered. Often it is necessary to determine how subsurface conditions or even the materials themselves will change over the design life of the project. Normally consolidated clays can gain strength with increases in effective stress and overconsolidated clays may lose strength with time when exposed in cuts. Some construction materials such as weak rock may loose strength due to weathering within the design life of the embankment.

5.3 Methods of Determining Soil and Rock Properties

Subsurface soil or rock properties are generally determined using one or more of the following methods:

- in-situ testing during the field exploration program,
- laboratory testing, and
- back analysis based on site performance data.

The two most common in-situ test methods for use in soil are the Standard Penetration Test, (SPT) and the cone penetrometer test (CPT). The laboratory testing program generally consists of index tests to obtain general information or to use with correlations to estimate design properties, and performance tests to directly measure specific engineering properties. The observational method, or use of back analysis, to determine engineering properties of soil or rock is often used with slope failures, embankment settlement or excessive settlement of existing structures. With landslides or slope failures, the process generally starts with determining the geometry of the failure and then determining the soil/rock parameters or subsurface conditions that cause the safety factor to approach 1.0. Often the determination of the properties is aided by correlations with index tests or experience on other projects. For embankment settlement, a range of soil properties is generally determined based on laboratory performance testing on undisturbed samples. Monitoring of fill settlement and pore pressure in the soil during construction allows the soil properties and prediction of the rate of future settlement to be refined. For structures such as bridges that experience unacceptable settlement or retaining walls that have excessive deflection, the engineering properties of the soils can sometimes be determined if the magnitudes of the loads are known. As with slope stability analysis, the geometry of the subsurface soil must be adequately known, including the history of the groundwater level at the site.

The detailed measurement and interpretation of soil and rock properties shall be consistent with the guidelines provided in FHWA-IF-02-034, *Evaluation of Soil and Rock Properties*, Geotechnical Engineering Circular No. 5 (**Sabatini, et al., 2002**), except as specifically indicated herein.

5.4 In-Situ Field Testing

Standards and details regarding field tests such as the Standard Penetration Test (SPT), the Cone Penetrometer Test (CPT), the vane shear test, and other tests and their use provided in **Sabatini, et al. (2002)** should be followed, except as specifically noted herein. Regarding Standard Penetration Tests (SPT), the N-values obtained are dependent on the equipment used and the skill of the operator, and should be corrected for field procedures to standard N_{60} values (an efficiency of 60 percent is typical for rope and cathead systems). This correction is necessary because many of the correlations developed to determine soil properties are based on N_{60}-values.

SPT N values should be corrected for hammer efficiency, if applicable to the design method or correlation being used, using the following relationship.

$$N_{60} = (ER/60\%)\,N \qquad (5\text{-}1)$$

Where:

N_{60} = SPT blow count corrected for hammer efficiency (blows/ft)

ER = Hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used.

The following values for ER may be assumed if hammer specific data are not available:

ER = 60% for conventional drop hammer using rope and cathead
ER = 80% for automatic trip hammer

Hammer efficiency (ER) for specific hammer systems used in local practice may be used in lieu of the values provided. If used, specific hammer system efficiencies shall be developed in general accordance with ASTM D-4945 for dynamic analysis of driven piles or other accepted procedure.

Corrections for rod length, hole size, and use of a liner may also be made if appropriate. In general, these are only significant in unusual cases or where there is significant variation from standard procedures. These corrections may be significant for evaluation of liquefaction. Information on these additional corrections may be found in: "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils"; Publication Number: MCEER-97-0022; T.L. Youd, I.M. Idriss (1997).

N-values are also affected by overburden pressure, and in general should be corrected for that effect, if applicable to the design method or correlation being used. N values corrected for both overburden and the efficiency of the field procedures used shall be designated as $N1_{60}$. The overburden correction equation that should be used is:

$$N1_{60} = C_N N_{60} \quad (5\text{-}2)$$

Where,

$$C_N = [0.77 \log_{10} (20/\sigma'_v)],\ C_N < 2.0 \quad (5\text{-}3)$$

C_N = correction factor for overburden
N_{60} = N-value corrected for energy efficiency
σ'_v = vertical effective stress at the location of the SPT N-value (TSF)

In general, correlations between N-values and soil properties should only be used for cohesionless soils, and sand in particular. Caution should be used when using N-values obtained in gravelly soil. Gravel particles can plug the sampler, resulting in higher blow counts and estimates of friction angles than actually exist. Caution should also be used when using N-values to determine silt or clay parameters due to the dynamic nature of the test and resulting rapid changes in pore pressures and disturbance within the deposit. Correlations of N-values with cohesive soil properties should generally be considered as preliminary. N-values can also be used for liquefaction analysis. See **WSDOT GDM Chapter 6** for more information regarding the use of N-values for liquefaction analysis.

In general design practice, hydraulic conductivity is estimated based on grain size characteristics of the soil strata (see Highway Runoff Manual M31-16, Section 4-5). In critical applications, the hydraulic conductivity may be determined through in-situ testing. A discussion of field measurement of permeability is presented in **Sabatini, et al. (2002)**, and ASTM D 4043 presents a guide for the selection of various field methods. If in-situ test methods are utilized to determine hydraulic conductivity, one or more of the following methods should be used:

- Well pumping tests
- Packer permeability tests
- Seepage Tests
- Slug tests
- Piezocone tests
- Flood tests or Pit Infiltration Tests (PIT) – applies mainly to infiltration facility design – see Section 4-5 of the **WSDOT Highway Runoff Manual (2004)**

5.4.1 Well Pumping Tests

Pump tests can be used to provide an estimate of the overall hydraulic conductivity of a geologic formation, and since it is in essence a full scale test, directly accounts for the layering and directionality of the hydraulic characteristics of the formation. The data provided can be used to determine the requirements for construction dewatering systems for excavations. However, pump tests can be quite expensive and can take a significant amount of time to complete. Furthermore, care must be exercised when conducting this type of test, especially if potentially contaminated zones are present that could be mobilized during pumping. This could also create problems with disposal of the pumped water. Impact to adjacent facilities such as drinking wells and subsidence caused by dewatering should be evaluated when planning this type of test. For this test, the method prescribed in ASTM D 4050 should be used. Analysis of the results of pumping tests requires experience and a thorough knowledge of the actual geologic conditions present at the test location. The time-drawdown response curves are unique to a particular geologic condition. Therefore, knowledge of the actual geologic conditions present at the test location is required in order to choose the correct analysis procedure, e.g., whether the aquifer is leaky, unconfined, or bounded, etc.

5.4.2 Packer Permeability Tests

Packer permeability tests can be used to measure the hydraulic conductivity of a specific soil or rock unit. The information obtained is used primarily in seepage studies. This test is conducted by inserting the packer units to the desired test location after the boring has been properly cleaned out. The packers are expanded to seal off the zone being tested, and water is injected into the borehole under constant pressure. Measurements of the flow rate are taken at regular time intervals. Upon completion of testing at a particular depth, the packers are lowered to a new test depth. Test depths should be determined from cores and geophysical logs of the borehole, prior to hydraulic conductivity testing. Note that if the packer test is performed in soil borings, casing must be installed.

5.4.3 Seepage Tests

Three types of seepage tests are commonly used: falling head, rising head and constant water level methods. In general, either the rising or falling level methods should be used if the hydraulic conductivity is low enough to permit accurate determination of the water level. In the falling head method, the borehole or piezometer is filled with water that is allowed to seep into the soil. The rate of drop of the water surface in the casing is monitored. The rising head method consists of bailing the water out of the borehole and observing the rate of rise until the change becomes negligible. The constant water level method is used if soil is too permeable to allow accurate measurement of the rising or falling water level. General guidance on these types of tests are provided in **Mayne, et al. (1997)**.

Boreholes (or in subsequently installed piezometers) in which seepage tests are to be performed should be drilled using only clear water as the drilling fluid. This precludes the formation of a mud cake on the walls of the hole or clogging of the soil pores with drilling mud. The tests can be performed intermittently as the borehole is advanced. In general, the rising head test is preferred because there is less chance of clogging soil pores with suspended sediment.

Data from seepage tests only reflect the hydraulic conditions near the borehole. In addition the actual area of seepage at the base of the borehole may not be accurately known. During the rising head test, there is the danger of the soil at the bottom of the borehole becoming loosened or “quick” if too great a gradient is imposed. However, seepage tests can be used in soils with lower hydraulic conductivities than is generally considered suitable for pumping tests and large volumes of water do not need to be disposed of. Also note that if the test is conducted inside the piezometer, the hydraulic conductivity measured from this could be influenced by the material placed inside the borehole around the screened pipe.

5.4.4 Slug Tests

These tests are easy to perform and can be performed in a borehole in which a screened pipe is installed. Two types of slug tests are commonly used, falling head and rising head. Falling head slug tests are conducted by lowering a solid object such as a weighted plastic cylinder into the borehole causing an instantaneous water level rise. As the water level gradually returns to static, the rate is recorded. A rising head slug test can then be performed by suddenly removing the slug, causing an instantaneous lowering of the water level. By monitoring the rate of rise or fall of the water level in the borehole, an estimate of the hydraulic conductivity can be determined. . For this test, the method prescribed in ASTM D 4044 should be used. However slug tests are not very reliable and may underestimate hydraulic conductivity by one or two orders of magnitude, particularly if the test well has been inadequately developed prior to testing. The test data will not provide an indication of the accuracy of the computed value unless a pumping test is done in conjunction with the slug test. Because the slug tests are short duration, they reflect hydraulic properties of the soil immediately surrounding the well intake.

5.4.5 Piezocone Tests

Details of the equipment and methodology used to conduct the piezocone test are provided in **Sabatini, et al. (2002)**. Piezocone data can be useful to estimate the hydraulic conductivity of silts and clays from interpretation of the coefficient of horizontal consolidation, c_h, obtained from the piezocone measurments. The procedure involves pushing the cone to the desired depth, followed by recording pore pressures while the cone is held stationary. The test is usually run until 50 percent of the excess pore pressure has dissipated (t_{50}). This requires knowledge of the initial in situ pore pressure at the test location. Dissipation tests are generally effective in silts and clays where large excess pore pressures are generated during insertion of the cone. Hydraulic conductivity can be estimated using various correlations with t_{50} and coefficient of horizontal consolidation (c_h), (see **Lunne, et al. (1997)**, and **Sabatini, et al. (2002)**). Estimation of hydraulic conductivity from CPT tests is subject to a large amount of uncertainty, and should be used as a preliminary estimate of permeability only.

5.4.6 Flood Tests

Flood tests or pilot infiltration tests are not always feasible, and in general are only used where unusual site conditions are encountered that are poorly modeled by correlation to soil gradation characteristics, and there is plenty of water available to conduct the test. The key to the success of this type of test is the estimate of the hydraulic gradient during the test, recognizing that the test hydraulic gradient could be much higher than the hydraulic gradient that is likely in service for the facility being designed. For more information, see the **WSDOT Highway Runoff Manual (2004)**.

5.5 Laboratory Testing of Soil and Rock

Laboratory testing is a fundamental element of a geotechnical investigation. The ultimate purpose of laboratory testing is to utilize repeatable procedures to refine the visual observations and field testing conducted as part of the subsurface field exploration program, and to determine how the soil or rock will behave under the imposed conditions. The ideal laboratory program will provide sufficient data to complete an economical design without incurring excessive tests and costs. Depending on the project issues, testing may range from simple soil classification testing to complex strength and deformation testing.

5.5.1 Quality Control for Laboratory Testing

Improper storage, transportation and handling of samples can significantly alter the material properties and result in misleading test results. The requirements provided in **WSDOT GDM Chapter 3** regarding these issues shall be followed.

Laboratories conducting geotechnical testing shall be either AASHTO accredited or fulfill the requirements of AASHTO R18 for qualifying testers and calibrating/verifications of testing equipment for those tests being performed. In addition, the following guidelines (**Mayne, et al., 1997**) for laboratory testing of soils should be followed:

1. Protect samples to prevent moisture loss and structural disturbance.
2. Carefully handle samples during extrusion of samples; samples must be extruded properly and supported upon their exit from the tube.
3. Avoid long term storage of soil samples in Shelby tubes.
4. Properly number and identify samples.
5. Store samples in properly controlled environments.
6. Visually examine and identify soil samples after removal of smear from the sample surface.
7. Use pocket penetrometer or miniature vane only for an indication of strength.
8. Carefully select "representative" specimens for testing.
9. Have a sufficient number of samples to select from.
10. Always consult the field logs for proper selection of specimens.
11. Recognize disturbances caused by sampling, the presence of cuttings, drilling mud or other foreign matter and avoid during selection of specimens.
12. Do not depend solely on the visual identification of soils for classification.
13. Always perform organic content tests when classifying soils as peat or organic. Visual classifications of organic soils may be very misleading.
14. Do not dry soils in overheated or underheated ovens.
15. Discard old worn-out equipment; old screens for example, particularly fine (< No. 40) mesh ones need to be inspected and replaced often, worn compaction mold or compaction hammers (an error in the volume of a compaction mold is amplified 30x when translated to unit volume) should be checked and replaced if needed.
16. Performance of Atterberg Limits requires carefully adjusted drop height of the Liquid Limit machine and proper rolling of Plastic Limit specimens.
17. Do not use tap water for tests where distilled water is specified.
18. Properly cure stabilization test specimens.
19. Never assume that all samples are saturated as received.

20. Saturation must be performed using properly staged back pressures.
21. Use properly fitted o-rings, membranes, etc. in triaxial or permeability tests.
22. Evenly trim the ends and sides of undisturbed samples.
23. Be careful to identify slickensides and natural fissures. Report slickensides and natural fissures.
24. Also do not mistakenly identify failures due to slickensides as shear failures.
25. Do not use unconfined compression test results (stress-strain curves) to determine elastic modulus values.
26. Incremental loading of consolidation tests should only be performed after the completion of each primary stage.
27. Use proper loading rate for strength tests.
28. Do not guesstimate e-log p curves from accelerated, incomplete consolidation tests.
29. Avoid "Reconstructing" soil specimens, disturbed by sampling or handling, for undisturbed testing.
30. Correctly label laboratory test specimens.
31. Do not take shortcuts: using non-standard equipment or non-standard test procedures.
32. Periodically calibrate all testing equipment and maintain calibration records.
33. Always test a sufficient number of samples to obtain representative results in variable material.

5.5.2 Developing the Testing Plan

The amount of laboratory testing required for a project will vary depending on availability of preexisting data, the character of the soils and the requirements of the project. Laboratory tests should be selected to provide the desired and necessary data as economically as possible. Specific geotechnical information requirements are provided in the WSDOT GDM chapters that address design of specific types of geotechnical features. Laboratory testing should be performed on both representative and critical test specimens obtained from geologic layers across the site. Critical areas correspond to locations where the results of the laboratory tests could result in a significant change in the proposed design. In general, a few carefully conducted tests on samples selected to cover the range of soil properties with the results correlated by classification and index tests is the most efficient use of resources.

The following should be considered when developing a testing program:

- Project type (bridge, embankment, rehabilitation, buildings, etc.)
- Size of the project
- Loads to be imposed on the foundation soils
- Types of loads (i.e., static, dynamic, etc.)
- Whether long-term conditions or short-term conditions are in view
- Critical tolerances for the project (e.g., settlement limitations)
- Vertical and horizontal variations in the soil profile as determined from boring logs and visual identification of soil types in the laboratory
- Known or suspected peculiarities of soils at the project location (i.e., swelling soils, collapsible soils, organics, etc.)

- Presence of visually observed intrusions, slickensides, fissures, concretions, etc in sample – how will it affect results
- Project schedules and budgets

Details regarding specific types of laboratory tests and their use are provided in **Sabatini, et al. (2002)**.

5.6 Engineering Properties of Soil

5.6.1 Laboratory Performance Testing

Laboratory performance testing is mainly used to estimate strength, compressibility, and permeability characteristics of soil and rock. For rock, the focus is typically on the shear strength of the intact rock, or on the shear strength of discontinuities (i.e., joint/seam) within the rock mass. For soil, shear strength may be determined on either undisturbed specimens of finer grained soil (undisturbed specimens of granular soils are very difficult, if not impossible, to get), or disturbed or remolded specimens of fine or coarse grained soil. There are a variety of shear strength tests that can be conducted, and the specific type of test selected depends on the specific application. See **Sabatini, et al. (2002)** for specific guidance on the types of shear strength tests needed for various applications, as well as the chapters in the WSDOT GDM that cover specific geotechnical design topics.

Disturbed soil shear strength testing is less commonly performed, and is primarily used as supplementary information when performing back-analysis of existing slopes, or for fill material and construction quality assurance when a minimum shear strength is required. It is difficult to obtain very accurate shear strength values through shear strength testing of disturbed (remolded) specimens since the in-situ density and soil structure is quite difficult to accurately recreate, especially considering the specific in-situ density may not be known. The accuracy of this technique in this case must be recognized when interpreting the results. However, for estimating the shear strength of compacted backfill, more accurate results can be obtained, since the soil placement method, as well as the in-situ density and moisture content, can be recreated in the laboratory with some degree of confidence. The key in the latter case is the specimen size allowed by the testing device, as in many cases, compacted fills have a significant percentage of gravel sized particles, requiring fairly large test specimens (i.e., minimum 3 to 4 inch diameter, or narrowest dimension specimens of 3 to 4 inches).

Typically, a disturbed sample of the granular backfill material (or native material in the case of obtaining supplementary information for back-analysis of existing slopes) is sieved to remove particles that are too large for the testing device and test standard, and is compacted into a mold to simulate the final density and moisture condition of the material. The specimens may or may not be saturated after compacting them and placing them in the shear testing device, depending on the condition that is to be simulated. In general, a drained test is conducted, or if it is saturated, the pore pressure during shearing can be measured (possible for triaxial testing; generally not possible for direct shear testing) to obtained drained shear strength parameters. Otherwise, the test is run slow enough to be assured that the specimen is fully drained during shearing (note that estimating the testing rate to assure drainage can be difficult). Multiple specimens tested using at least three confining pressures should be tested to obtain a shear strength envelope. See **Sabatini, et al. (2002)** for additional details.

Tests to evaluate compressibility or permeability of existing subsurface deposits must be conducted on undisturbed specimens, and the less disturbance the better. See **Sabatini, et al. (2002)** for additional requirements regarding these and other types of laboratory performance tests that should be followed.

The hydraulic conductivity of a soil is influenced by the particle size and gradation, the void ratio, mineral composition, and soil fabric. In general the hydraulic conductivity, or permeability, increases with increasing grain-size; however, the size and shape of the voids also has a significant influence. The smaller the voids, the lower the permeability. Mineral composition and soil fabric have little effect on the permeability of gravel, sand, and non-plastic silt, but are important for plastic silts and clays. Therefore, relationships between particle size and permeability are available for coarse-grained materials, some of which are presented in the Correlations subsection (**WSDOT GDM Section 5.6.2**). In general, for clays, the lower the ion exchange capacity of the soil, the higher the permeability. Likewise, the more flocculated (open) the structure, the higher the permeability.

The methods commonly used to determine the hydraulic conductivity in the laboratory include, the constant head test, the falling head test, and direct or indirect methods during a consolidation test. The laboratory tests for determining the hydraulic conductivity are generally considered quite unreliable. Even with considerable attention to test procedures and equipment design, tests may only provide values within an order of magnitude of actual conditions. Some of the factors for this are:

- The soil in-situ is generally stratified and this is difficult to duplicate in the laboratory.
- The horizontal value of k is usually needed, but testing is usually done on tube samples with vertical values obtained.
- In sand, the horizontal and vertical values of k are significantly different, often on the order of k_h = 10 to 100k_v.
- The small size of laboratory samples leads to boundary condition effects.
- Saturated steady state soil conditions are used for testing, but partially saturated soil water flow often exists in the field.
- On low permeability soils, the time necessary to complete the tests causes evaporation and equipment leaks to be significant factors.
- The hydraulic gradient in the laboratory is often 5 or more to reduce testing time, whereas in the field it is more likely in the range of 0.1 to 2.

The hydraulic conductivity is expected to vary across the site; however, it is important to differentiate errors from actual field variations. When determining the hydraulic conductivity, the field and laboratory values should be tabulated along with the other known data such as sample location, soil type, grain-size distribution, Atterberg limits, water content, stress conditions, gradients, and test methods. Once this table is constructed, it will be much easier to group like soil types and k values to delineate distinct areas within the site, and eliminate potentially erroneous data.

5.6.2 Correlations to Estimate Engineering Properties of Soil

Correlations that relate in-situ index test results such as the SPT or CPT or laboratory soil index testing may be used in lieu of or in conjunction with performance laboratory testing and back-analysis of site performance data to estimate input parameters for the design of the geotechnical elements of a project. Since properties estimated from correlations tend to have greater variability than measurement using laboratory performance data (see **Phoon, et al., 1995**), properties estimated from correlation to in-situ field index testing or laboratory index testing should be based on multiple measurements within each significant geologic unit (if the geologic unit is large enough to obtain multiple measurements). A minimum of 3 to 5 measurements should be obtained from each geologic unit as the basis for estimating design properties.

The drained friction angle of granular deposits should be determined based on the correlation provided in **Table 5-1**.

Table 5-1. Correlation of SPT N values to drained friction angle of granular soils (**modified after Bowles, 1977**).

$N1_{60}$ from SPT (blows/ft)	ϕ (°)
<4	25-30
4	27-32
10	30-35
30	35-40
50	38-43

The correlation used is modified after **Bowles (1977)**. The correlation of **Peck, Hanson and Thornburn (1974)** falls within the ranges specified. Experience should be used to select specific values within the ranges. In general, finer materials or materials with significant silt-sized material will fall in the lower portion of the range. Coarser materials with less then 5% fines will fall in the upper portion of the range.

Care should be exercised when using other correlations of SPT results to soil parameters. Some published correlations are based on corrected values ($N1_{60}$) and some are based on uncorrected values (N). The designer should ascertain the basis of the correlation and use either $N1_{60}$ or N as appropriate. Care should also be exercised when using SPT blow counts to estimate soil shear strength if in soils with coarse gravel, cobbles, or boulders. Large gravels, cobbles, or boulders could cause the SPT blow counts to be unrealistically high.

Correlations for other soil properties (other than as specifically addressed above for the soil friction angle) as provided in **Sabatini, et al. (2002)** may be used if the correlation is well established and if the accuracy of the correlation is considered regarding its influence if the estimate obtained from the correlation in the selection of the property value used for design. Local geologic formation-specific correlations may also be used if well established by data comparing the prediction from the correlation to measured high quality laboratory performance data, or back-analysis from full scale performance of geotechnical elements affected by the geologic formation in question.

Regarding soil hydraulic conductivity, the correlation provided in the ***WSDOT Highway Runoff Manual,*** Section 4-5 should be used.

5.7 Engineering Properties of Rock

Engineering properties of rock are generally controlled by the discontinuities within the rock mass and not the properties of the intact material. Therefore, engineering properties for rock must account for the properties of the intact pieces and for the properties of the rock mass as a whole, specifically considering the discontinuities within the rock mass. A combination of laboratory testing of small samples, empirical analysis, and field observations should be employed to determine the engineering properties of rock masses, with greater emphasis placed on visual observations and quantitative descriptions of the rock mass.

Rock properties can be divided into two categories: intact rock properties and rock mass properties. Intact rock properties are determined from laboratory tests on small samples typically obtained from coring, outcrops or exposures along existing cuts. Engineering properties typically obtained from laboratory tests include specific gravity, unit weight, ultrasonic velocity, compressive strength, tensile strength, and shear strength. Rock mass properties are determined by visual examination of discontinuities within the rock mass, and how these discontinuities will affect the behavior of the rock mass when subjected to the proposed construction.

The methodology and related considerations provided by **Sabatini, et al. (2002)** should be used to assess the design properties for the intact rock and the rock mass as a whole. However, the portion of **Sabatini, et al. (2002)** that addresses the determination of fractured rock mass shear strength parameters (**Hoek and Brown, 1988**) is outdated. The original work by Hoek and Brown has been updated and is described in **Hoek, et al. (2002)**. The updated method uses a Geological Strength Index (GSI) to characterize the rock mass for the purpose of estimating strength parameters, and has been developed based on re-examination of hundreds of tunnel and slope stability analyses in which both the 1988 and 2002 criteria were used and compared to field results. While the 1988 method has been more widely published in national (e.g., FHWA) design manuals than has the updated approach provided in **Hoek, et al. (2002)**, considering that the original developers of the method have recognized the short-comings of the 1988 method and have reassessed it through comparison to actual rock slope stability data, WSDOT considers the **Hoek, et al. (2002)** to be the most accurate methodology. Therefore the **Hoek, et al. (2002)** method should be used for fractured rock mass shear strength determination. Note that this method is only to be used for highly fractured rock masses in which the stability of the rock slope is not structurally controlled. See **WSDOT GDM Chapter 12** for additional requirements regarding the assessment of rock mass properties.

5.8 Final Selection of Design Values

5.8.1 Overview

After the field and laboratory testing is completed, the geotechnical designer should review the quality and consistency of the data, and should determine if the results are consistent with expectations. Once the lab and field data have been collected, the process of final material property selection begins. At this stage, the geotechnical designer generally has several sources of data consisting of that obtained in the field, laboratory test results and correlations from index testing. In addition, the geotechnical designer may have experience based on other projects in the area or in similar soil/rock conditions. Therefore, if the results are not consistent with each other or previous experience, the reasons for the differences should be evaluated, poor data eliminated and trends in data identified. At this stage it may be necessary to conduct additional performance tests to try to resolve discrepancies.

As stated in **WSDOT GDM Section 5.1**, the focus of geotechnical design property assessment and final selection is on the individual geologic strata identified at the project site. A geologic stratum is characterized as having the same geologic depositional history and stress history, and generally has similarities throughout the stratum in its density, source material, stress history, and hydrogeology. All of the information that has been obtained up to this point including preliminary office and field reconnaissance, boring logs, CPT soundings etc., and laboratory data are used to determine soil and rock engineering properties of interest and develop a subsurface model of the site to be used for design. Data from different sources of field and lab tests, from site geological characterization of the site subsurface conditions, from visual observations obtained from the site reconnaissance, and from historical experience with the subsurface conditions at or near the site must be combined to determine the engineering properties for the various geologic units encountered throughout the site. However, soil and rock properties for design should not be averaged across multiple strata, since the focus of this property characterization is on the individual geologic stratum. Often, results from a single test (e.g. SPT N-values) may show significant scatter across a site for a given soil/rock unit. Perhaps data obtained from a particular soil unit for a specific property from two different tests (e.g. field vane shear tests and lab UU tests) do not agree. Techniques should be employed to determine the validity and reliability of the data and its usefulness in selecting final design parameters. After a review of data reliability, a review of the variability of the selected parameters should be carried out. Variability can manifest itself in two ways: 1) the inherent in-situ variability of a particular parameter due to the variability of the soil unit itself, and 2) the variability associated with estimating the parameter from the various testing methods. From this step final selection of design parameters can commence, and from there completion of the subsurface profile.

5.8.2 Data Reliability and Variability

Inconsistencies in data should be examined to determine possible causes and assess any mitigation procedures that may be warranted to correct, exclude, or downplay the significance of any suspect data. The following procedures provide a step-by-step method for analyzing data and resolving inconsistencies as outlined by **Sabatini, et al. (2002)**:

1) **Data Validation:** Assess the field and the laboratory test results to determine whether the reported test results are accurate and are recorded correctly for the appropriate material. For lab tests on undisturbed samples consider the effects of sample disturbance on the quality of the data. For index tests (e.g. grain size, compaction) make sure that the sample accurately represents the in situ condition. Disregard or downplay potentially questionable results.
2) **Historical Comparison:** Assess results with respect to anticipated results based on site and/or regional testing and geologic history. If the new results are inconsistent with other site or regional data, it will be necessary to assess whether the new data is anomalous or whether the new site conditions differ from those from which previous data was collected. For example, an alluvial deposit might be expected to consist of medium dense silty sand with SPT blow counts less than 30. If much higher blow counts are recorded, the reason could be the deposit is actually dense (and therefore higher friction angles can be assumed), or gravel may be present and is influencing the SPT data. Most likely it is the second case, and the engineering properties should probably be adjusted to account for this. But if consideration had not been given as to what to expect, values for properties might be used that could result in an unconservative design.
3) **Performance Comparison:** Assess results with respect to historic performance of structures at the site or within similar soils. Back analysis of previous landslides and retaining wall movement in the same geologic units under consideration, if available, should be performed to estimate shear strength parameters. Settlement data from existing embankments, if available, should be used to estimate compressibility and settlement rates. Results can be compared to the properties determined from field and lab testing for the project site. The newly collected data can be correlated with the parameters determined from observation of performance and the field and lab tests performed for the previous project.
4) **Correlation Calibration:** If feasible, develop site-specific correlations using the new field and lab data. Assess whether this correlation is within the range of variability typically associated with the correlation based on previous historic data used to develop the generic correlation.
5) **Assess Influence of Test Complexity:** Assess results from the perspective of the tests themselves. Some tests may be easy to run and calibrate, but provide data of a “general” nature while other tests are complex and subject to operator influence, yet provide “specific” test results. When comparing results from different tests consider which tests have proven to give more accurate or reliable results in the past, or more accurately approximate anticipated actual field conditions. For example, results

of field vane shear tests may be used to determine undrained shear strength for deep clays instead of laboratory UU tests because of the differences in stress states between the field and lab samples. It may be found that certain tests consistently provide high or low values compared to anticipated results.

The result of these steps is to determine whether or not the data obtained for the particular tests in question is valid. Where it is indicated that test results are invalid or questionable, the results should be downplayed or thrown out. If the test results are proven to be valid, the conclusion can be drawn that the soil unit itself and its corresponding engineering properties are variable (vertically, aerially, or both).

The next step is to determine the amount of variability that can be expected for a given engineering property in a particular geologic unit, and how that variability should influence the selection of the final design value. **Sabatini, et al. (2002)** list several techniques that can be used:

1) **Experience:** In some cases the geotechnical designer may have accumulated extensive experience in the region such that it is possible to accurately select an average, typical or design value for the selected property, as well as the appropriate variability for the property.
2) **Statistics:** If a geotechnical designer has extensive experience in a region, or there has been extensive testing by others with published or available results, there may be sufficient data to formally establish the average value and the variability (mean and standard deviation) for the specific property. See **Sabatini, et al. (2002)** and **Phoon, et al. (1995)** for information on the variability associated with various engineering properties.
3) **Establish Best-Case and Worst-Case Scenarios:** Based on the experience of the geotechnical designer, it may be possible to establish upper and lower bounds along with the average for a given property.

5.8.3 Final Property Selection

The final step is to incorporate the results of the previous section into the selection of design values for required design properties.

Recognizing the variability discussed in the previous section, depending on the amount of variability estimated or measured, the potential impact of that variability (or uncertainty) on the level of safety in the design should be assessed. If the impact of this uncertainty is likely to be significant, parametric analyses should be conducted, or more data could be obtained to help reduce the uncertainty. Since the sources of data that could be considered may include both measured laboratory data, field test data, performance data, and other previous experience with the geologic unit(s) in question, it will not be possible to statistically combine all this data together to determine the most likely property value. Engineering judgment, combined with parametric analyses as needed, will be needed to make this final assessment and design property determination. At that point, a decision must be made as to whether the final design value selected should reflect the interpreted average value for the property, or a value that is somewhere between the most likely average value and the most conservative estimate of the property. However, the desire for design safety must be balanced with the cost effectiveness and constructability of the design. In some cases, being too conservative with the design could result in an un-constructible design (e.g., the use of very conservative design parameters could result in a pile foundation that must be driven deep into a very dense soil unit that in reality is too dense to penetrate with available equipment).

Note that in **WSDOT GDM Chapter 8**, where reliability theory was used to establish load and resistance factors, the factors were developed assuming that mean values for the design properties are used. However, even in those cases, design values that are more conservative than the mean may still be appropriate, especially if there is an unusual amount of uncertainty in the assessment of the design properties due, for example to highly variable site conditions, lack of high quality data to assess property values, or due to widely divergent property values from the different methods used to assess properties within a given geologic unit. Depending on the availability of soil or rock property data and the variability of the geologic strata under consideration, it may not be possible to reliably estimate the average value of the properties needed for design. In such cases, the geotechnical designer may have no choice but to use a more conservative selection of design parameters to mitigate the additional risks created by potential variability or the paucity of relevant data. Note that for those resistance factors that were determined based on calibration by fitting to allowable stress design, this property selection issue is not relevant, and property selection should be based on the considerations discussed previously.

The process and examples to make the final determination of properties to be used for design provided by **Sabatini, et al. (2002)** should be followed.

5.8.4 Development of the Subsurface Profile

While **WSDOT GDM Section 5.8** generally follows a sequential order, it is important to understand that the selection of design values and production of a subsurface profile is more of an iterative process. The development of design property values should begin and end with the development of the subsurface profile. Test results and boring logs will likely be revisited several times as the data is developed and analyzed before the relation of the subsurface units to each other and their engineering properties are finalized.

The ultimate goal of a subsurface investigation is to develop a working model that depicts major subsurface layers exhibiting distinct engineering characteristics. The end product is the subsurface profile, a two dimensional depiction of the site stratigraphy. The following steps outline the creation of the subsurface profile:

1) Complete the field and lab work and incorporate the data into the preliminary logs.
2) Lay out the logs relative to their respective field locations and compare and match up the different soil and rock units at adjacent boring locations, if possible. However, caution should be exercised when attempting to connect units in adjacent borings, as the geologic stratigraphy does not always fit into nice neat layers. Field descriptions and engineering properties will aid in the comparisons.
3) Group the subsurface units based on engineering properties.
4) Create cross sections by plotting borings at their respective elevations and positions horizontal to one another with appropriate scales. If appropriate, two cross sections should be developed that are at right angles to each other so that lateral trends in stratigraphy can be evaluated when a site contains both lateral and transverse extents (i.e. a building or large embankment).
5) Analyze the profile to see how it compares with expected results and knowledge of geologic (depositional) history. Have anomalies and unexpected results encountered during exploration and testing been adequately addressed during the process? Make sure that all of the subsurface features and properties pertinent to design have been addressed.

5.8.5 Selection of Design Properties for Engineered Materials

This section provides guidelines for the selection of properties that are commonly used on WSDOT projects such as engineered fills. The engineering properties are based primarily on gradation and compaction requirements, with consideration of the geologic source of the fill material typical for the specific project location. For materials such as common borrow where the gradation specification is fairly broad, a wider range of properties will need to be considered.

Common Borrow. Per the WSDOT Standard Specifications, common borrow may be virtually any soil or aggregate either naturally occurring or processed which is substantially free of organics or other deleterious material, and is non-plastic. The specification allows for the use of more plastic common borrow when approved by the engineer. On WSDOT projects this material will generally be placed at 90 percent (Method B) or 95 percent (Method C) of Standard Proctor compaction. Because of the variability of the materials that may be used as common borrow, the estimation of an internal friction angle and unit weight should be based on the actual material used. A range of values for the different material properties is given in **Table 5-2**. Lower range values should be used for finer grained materials compacted to Method B specifications. In general during design, the specific source of borrow is not known. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the geotechnical designer has specific knowledge of the source(s) likely to be used, or unless quality assurance shear strength testing is conducted during construction. Common borrow will likely have a high enough fines content to be moderately to highly moisture sensitive. This moisture sensitivity may affect the design property selection if it is likely that placement conditions are likely to be marginal due to the timing of construction.

Select Borrow. The requirements for select borrow ensure that the mixture will be granular and contain at least a minimal amount of gravel size material. The materials are likely to be poorly graded sand and contain enough fines to be moderately moisture sensitive (the specification allows up to 10 percent fines). Select Borrow is not an all weather material. Triaxial or direct shear strength testing on material that meets Select Borrow gradation requirements indicates that drained friction angles of 38 to 45 degrees are likely when the soil is well compacted. Even in it loosest state, shear strength testing of relatively clean sands meeting Select Borrow requirements has indicated values of 30 to 35 degrees are likely. However, these values are highly dependent on the geologic source of the material. Much of the granular soil in Washington has been glacially derived, resulting in subangular to angular soil particles and hence, high shear strength values. Windblown, beach, or alluvial sands that have been rounded through significant transport could have significantly lower shear strength values. Left-overs from processed materials (e.g., scalpings) could also have relative low friction angles depending on the uniformity of the material and the degree of rounding in the soil particles. A range of values for shear strength and unit weight based on previous experience for well compacted Select Borrow is provided in **Table 5-2**. In general during design the specific source of borrow is not known. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the geotechnical designer has specific knowledge of the source(s) likely to be used or unless quality assurance shear strength testing is conducted during construction. Select Borrow with significant fines content may sometimes be modeled as having a temporary or apparent cohesion value from 50 to 200 psf. If a cohesion value is used, the friction angle should be reduced so as not to increase the overall strength of the material. For long term analysis, all the borrow material should be modeled with no cohesive strength.

Gravel Borrow – The gravel borrow specification should ensure a reasonably well graded sand and gravel mix. Because the fines content is under 7 percent, the material is only slightly moisture sensitive. However, in very wet conditions, material with lower fines content should be used. Larger diameter triaxial shear strength testing performed on well graded mixtures of gravel with sand that meet the Gravel Borrow specification indicate that very high internal angles of friction are possible, approaching 50 degrees, and that shear strength values less than 40 degrees are not likely. However, lower shear strength values are possible for Gravel Borrow from naturally occurring materials obtained from non-glacially derived sources such as wind blown or alluvial deposits. In many cases, processed materials are used for Gravel Borrow, and in general, this processed material has been crushed, resulting in rather angular particles and very high soil friction angles. Its unit weight can approach that of concrete if very well graded. A range of values for shear strength and unit weight based on previous experience is provided in **Table 5-2**. In general during design the specific source of borrow is not known. Therefore, it is not prudent to select a design friction angle that is near or above the upper end of the range unless the geotechnical designer has specific knowledge of the source(s) likely to be used or unless quality assurance shear strength testing is conducted during construction.

Backfill for Walls. Gravel backfill for walls is a free draining material that is generally used to facilitate drainage behind retaining walls. This material has similarities to Gravel Borrow, but generally contains fewer fines and is freer draining. Gravel backfill for Walls is likely to be a processed material and if crushed is likely to have a very high soil friction angle. A likely range of material properties is provided in **Table 5-2**.

Table 5-2. Presumptive design property ranges for borrow and other WSDOT Standard Specification materials.

Material	WSDOT Standard Specification	Soil Type (USCS classification)	ϕ (degrees)	Cohesion (psf)	Total Unit Weight (pcf)
Common Borrow	9-03.14(3)	ML, SM, GM	30 to 34	0	115 to 130
Select Borrow	9-03.14(2)	GP, GP-GM, SP, SP-SM	34 to 38	0	120 to 135
Gravel Borrow	9-03.14(1)	GW, GW-GM, SW, SW-SM	36 to 40	0	130 to 145
Gravel Backfill for Walls	9-03.12(2)	GW, GP, SW, SP	36 to 40	0	125 to 135

Rock embankment. Embankment material is considered rock embankment if 25 percent of the material is over 4 inches in diameter. Compactive effort is based on a method specification. Because of the nature of the material, compaction testing is generally not feasible. The specification allows for a broad range of material and properties such that the internal friction angle and unit weight can vary considerably based on the amount and type of rock in the fill. Rock excavated from cuts consisting of siltstone, sandstone and claystone may break down during the compaction process, resulting in less coarse material. Also, if the rock is weak, failure may occur through the rock fragments rather than around them. In these types of materials, the strength parameters may resemble those of earth embankments. For existing embankments, the soft rock may continue to weather with time, if the embankment materials continue to become wet. For sound rock embankments, the strength parameters may be much higher. For compacted earth embankments with sound rock, internal friction angles of up to 45 degrees may be reasonable. Unit weights for rock embankments generally range from 130 to 140 pcf.

Quarry Spalls and Rip Rap. Quarry spalls, light loose rip rap and heavy loose rip rap created from shot rock are often used as fill material below the water table or in shear keys in slope stability and landslide mitigation applications. WSDOT Standard Specification Section 9-13 provides minimum requirements for degradation and specific gravity for these materials. Therefore sound rock must be used for these applications. For design purposes, typical values of 120 to 130 pcf for the unit weight (this considers the large amount of void space due to the coarse open gradation of this type of material) and internal angles of friction of about 40 to 45 degrees should be used.

Wood Fiber. Wood fiber fills have been used by WSDOT for over 30 years in fill heights up to about 40 feet. The wood fiber has generally been used as light weight fill material over soft soil to improve embankment stability. Wood fiber has also been used in emergency repair because rain and wet weather does not affect the placement and compaction of the embankment. Only fresh wood fiber should be used to prolong the life of the fill, and the maximum particle size should be 6 inches or less. The wood fiber is generally compacted in lifts of about 12 inches with two passes of a track dozer. Presumptive design values of 50 pcf for unit weight and an internal angle of friction of about 40 degrees may be used for the design of the wood fiber fills **(Allen et al., 1993)**.

To mitigate the effects of leachate, the amount of water entering the wood should be minimized. Generally topsoil caps of about 2 feet in thickness are used. The pavement section should be a minimum of 2 feet (a thicker section may be needed depending on the depth of wood fiber fill). Wood fiber fill will experience creep settlement for several years and some pavement distress should be expected during that period. Additional information on the properties and durability of wood fiber fill is provided in **Kilian and Ferry (1993)**.

Geofoam. Geofoam has been used as lightweight fill on WSDOT projects for about 10 years. Geofoam ranges in unit weight from about 1 to 2 pcf. Geofoam constructed from expanded polystyrene (EPS) is manufactured according to ASTM standards for minimum density (ASTM C 303), compressive strength (ASTM D 1621) and water absorption (ASTM C 272). Type I and II are generally used in highway applications. Bales of recycled industrial polystyrene waste are also available. These bales have been used to construct temporary haul roads over soft soil. However, these bales should not be used in permanent applications.

5.9 Properties of Predominant Geologic Units in Washington

This section contains a brief discussion of soil and rock types common to Washington state that have specific engineering properties that need consideration.

5.9.1 Loess

Loess is a windblown (eolian) soil consisting mostly of silt with minor amounts of sand and clay (**Higgens et al., 1987**). Due to its method of deposition, loess has an open (honeycomb) structure with very high void ratios. The clay component of loess plays a pivotal role because it acts as a binder (along with calcium carbonate in certain deposits) holding the structure together. However, upon wetting, either the water soluble calcium carbonate bonds dissolve or the large negative pore pressures within the clay that are holding the soil together are reduced and the soil can undergo shear failures and/or settlements.

Loess deposits encompass a large portion of southeastern Washington. Loess typically overlies portions of the Columbia River Basalt Group and is usually most pronounced at the tops of low hills and plateaus where erosion has been minimal (**Joseph, 1990**). Washington loess has been classified into four geologic units: Palouse Loess, Walla Walla Loess, Ritzville Loess, and Nez Perce Loess. However, these classifications hold little relevance to engineering behavior. For engineering purposes loess can generally be classified into three categories based on grain size: clayey loess, silty loess, and sandy loess (see **WSDOT GDM Chapter 10**).

Typical index and performance properties measured in loess are provided in **Table 5-3**, based on the research results provided in Report WA-RD 145.2 (**Higgins and Fragaszy, 1988**). Density values typically increase from west to east across the state with corresponding increase in clay content. Higgins and Fragaszy observed that densities determined from Shelby tube samples in loess generally result in artificially high values due to disturbance of the open soil structure and subsequent densification. Studies of shear strength on loess have indicated that friction angles are usually fairly constant for a given deposit and are typically within the range of 27 to 29 degrees using CU tests. These studies have also indicated that cohesion values can be quite variable and depend on the degree of consolidation, moisture content and amount of clay binder. Research has shown that at low confining pressures, loess can lose all shear strength upon wetting.

Table 5-3. Typical measured properties for loess deposits in Washington state.

Type of Loess	Liquid Limit	Plasticity Index	Dry Density (pcf)	Angle of Internal Friction (^{o})
Clayey	33 to 49	11 to 27	70 to 90, with maximum of up to 95 to 98 (generally increases with clay content)	27 to 29 from CU tests
Silty	14 to 32	0 to 11		
Sandy	Nonplastic	Nonplastic		

The possibility of wetting induced settlements should be considered for any structure supported on loess by performing collapse tests. Collapse tests are usually performed as either single ring (ASTM D 5333) or double ring tests. Double ring tests have the advantage in that potential collapse can be estimated for any stress level. However, two identical samples must be obtained for testing. Single ring tests have the advantage in that they more closely simulate actual collapse conditions and thus give a more accurate estimate of collapse potential. However, collapse potential can only be estimated for a particular stress level, so care must be taken to choose an appropriate stress level for sample inundation during a test. When designing foundations in loess, it is important to consider long term conditions regarding possible changes in moisture content throughout the design life of the project. Proper drainage design is crucial to keeping as much water as possible from infiltrating into the soil around the structure. A possible mitigation technique could include overexcavation and recompaction to reduce or eliminate the potential for collapse settlement.

Loess typically has low values of permeability and infiltration rates. When designing stormwater management facilities in loess, detention ponds should generally be designed for very low infiltration rates.

Application of the properties of loess to cut slope stability is discussed in **WSDOT GDM Chapter 10**.

5.9.2 Peat/Organic Soils

Peats and organic soils are characterized by very low strength, very high compressibility (normally or slightly under-consolidated) and having very important time-consolidation effects. Often associated with wetlands, ponds and near the margins of shallow lakes, these soils pose special challenges for the design of engineering transportation projects. Deep deposits (+100 FT in some cases) with very high water content, highly compressibility, low strength and local high groundwater conditions require careful consideration regarding settlement and stability of earth fill embankments, support for bridge foundations, and locating culverts.

The internal structure of peat, either fibrous or granular, affects its capacity for retaining and releasing water and influences its strength and performance. With natural water content often ranging from 200-600 percent (over 100 for organic silts and sands) and wet unit weight ranging from 70 to 90 pcf, it can experience considerable shrinkage (>50%) when its dried out. Rewetting usually cannot restore its original volume or moisture content. Under certain conditions, dried peat will oxidize and virtually disappear. Undisturbed sampling for laboratory testing is difficult. Field vane testing is frequently used to evaluate in place shear strength, though in very fibrous peats, reliable shear strength data is difficult to obtain even with the field vane shear test. Initial undisturbed values of 100-400 PSF are not uncommon but remolded (residual) strengths can be 30 to 50 % less **(Schmertmann, 1967)**. Vane strength however, is a function of both vane size and peat moisture content. Usually, the lower the moisture of the peat and the greater its depth, the higher is its strength. Strength increases significantly when peat is consolidated, and peak strength only develops after large deformation has taken place. Due to the large amount of strain that can occur when embankment loads are placed on peats and organic soils, residual strengths may control the design.

Vertical settlement is also a major concern for constructing on organic soils. The amount of foundation settlement and the length of time for it to occur are usually estimated from conventional laboratory consolidation tests. Secondary compression can be quite large for peats and must always be evaluated when estimating long-term settlement. Based on experience in Washington state, compression index values based on vertical strain ($C_{c\varepsilon}$) typically range from 0.1 to 0.3 for organic silts and clays, and are generally above 0.3 to 0.4 for peats. The coefficient of secondary compression ($C_{\alpha\varepsilon}$) is typically equal to $0.05C_{c\varepsilon}$ to $0.06C_{c\varepsilon}$ for organic silts and peats, respectively.

5.9.3 Glacial Till and Glacial Advance Outwash

Glacial till typically consists of non-stratified deposits of clay, silt, sand and gravel with cobbles and occasional boulders. Although the matrix proportions of silt and clay vary from place to place, the matrix generally consists of silty sand or sandy silt (**Troost and Booth, 2003**). The glacial till has been glacially overridden, but the upper 2 to 5 feet is often weathered and is typically medium dense to dense. The glacial till generally grades to dense to very dense below the weathered zone.

Glacial till is often found near the surface in the Puget Sound Lowland area. The Puget Sound Lowland is a north-south trending trough bordered by the Cascade Mountains to the east and the Olympic Mountains to the west. The most recent glaciation, the Vashon Stade of the Fraser Glaciation occurred between roughly 18,000 to 13,000 years ago. Glacial till deposited by this glaciation extends as far south as the Olympia area.

Glacial till generally provides good bearing resistance because of its dense nature. Typically values used in slope stability evaluations range from 40 to 45 degrees for internal friction angle with cohesion values of 100 to 1,000 psf. Unit weights used for design are typically in the range of 130 to 140 pcf. The dense nature of glacial till tends to make excavation and sheet pile installation difficult. It is not uncommon to have to rip glacial till with a dozer or utilize large excavation equipment. Permeability in glacial till is relatively low because of the fines content and the density. However, localized pockets and seams of sand with higher permeability are occasional encountered in glacial till units.

Wet weather construction in glacial till is often difficult because of the relatively high fines content of glacial till soils. When the moisture content of these soils is more than a few percent above the optimum moisture content, glacial till soils become muddy and unstable, and operation of equipment on these soils can become difficult.

Glacial advance outwash is similar in nature to glacial till, but tends to be more coarse grained and cleaner (fewer fines). Properties are similar, but cohesion is lower, causing this material to have greater difficulty standing without raveling in a vertical cut, and in general can more easily cave in open excavations or drilled holes. Since it contains less fines, it is more likely to have relatively high permeability and be water bearing. In very clean deposits, non-displacement type piles (e.g., H-piles) can "run".

For both glacial till and glacial advance outwash, cobble and boulder sized material can be encountered any time. Boulders in these deposits can range from a foot or two in diameter to the size of a bus. In some areas they can also be nested together, making excavation very difficult.

5.9.4 Colluvium/Talus

Colluvium is a general term used to describe soil and rock material that has been transported and deposited by gravitational forces. Colluvium is typified by poorly sorted mixtures of soil and rock particles ranging in size from clay to large boulders. Talus is a special type of colluvium and refers to a particular landform and the material that comprises it. Talus deposits are generally made up of rock fragments of any size and shape (but usually coarse and angular) located at the base of cliffs and steep slopes.

Colluvium is a very common deposit, encompassing upwards of 90 percent of the ground surface in mountainous areas. Colluvial deposits are typically shallow (less than about 25 to 30 feet thick), with thickness increasing towards the base of slopes. Colluvium generally overlies bedrock along the slopes of hills and mountains and intermixes with alluvial material in stream bottoms.

Subsurface investigations in colluvium using drilling equipment are usually difficult because of the heterogeneity of the deposit and possible presence of large boulders. In addition, site access and safety issues also can pose problems. Test pits and trenches offer alternatives to conventional drilling that may provide better results. Subsurface investigations in talus are usually even more difficult, if not impossible. Engineering properties of talus are extremely difficult to determine in the lab or in situ. A useful method for determining shear strength properties in both colluvium and talus is to analyze an existing slope failure. For talus, this may be the only way to estimate shear strength parameters. Talus deposits can be highly compressible because of the presence of large void spaces. Colluvium and talus slopes are generally marginally stable. In fact, talus slopes are usually inclined at the angle of repose of the constituent material. Cut slopes in colluvium often result in steepened slopes beyond the angle of repose, resulting in instability. Slope instability is often manifested by individual rocks dislodging from the slope face and rolling downslope. While the slope remains steeper than the angle of repose a continuous and progressive failure will occur.

Construction in colluvium is usually difficult because of the typical heterogeneity of deposits and corresponding unfavorable characteristics such as particle size and strength variations and large void spaces. In addition, there is the possibility of long term creep movement. Large settlements are also possible in talus. Foundations for structures in talus should extend through the deposit and bear on more competent material. Slope failures in colluvium are most often caused by infiltration of water from intense rainfall. Modifications to natural slopes in the form of cut slopes and construction of drainage ditches are one way that water can infiltrate into a colluvial soil and initiate a slope failure. Careful consideration must be given to the design of drainage facilities to prevent the increase of water in colluvial soils.

5.9.5 Columbia River Sand

These sands are located in the Vancouver area and may have been deposited by backwaters from the glacial Lake Missoula catastrophic floods. The sands are poorly graded and range from loose to medium dense. The sand is susceptible to liquefaction if located below the water table. The sands do not provide a significant amount of frictional resistance for piles, and non-displacement piles may tend to run. Based on the observed stability of slopes in this formation, soil friction angles of 28^{o} to 32^{o} should be expected.

5.9.6 Columbia Basin Basalts

The basalt flows that dominate the Columbia Basin were erupted into a structural and topographic low between the northern Rocky Mountains and the rising Cascade Range. During periods between the flows, erosion would take place and tuffs, sandstones, and conglomerates would be deposited on top of basalt flows **(Thorsen, 1989)**. In some areas lake beds formed. The resulting drainage systems and lakes were responsible for the extensive layer of sediments between, interfingering with, and overlying the basalt flows. These sediments are generally thicker in areas peripheral to the flows, especially in and along the western part of the basin. During the interludes between flows, deep saprolites formed on some flow surfaces. Present topographic relief on the basin has been provided largely by a series of east-west trending anticlinal folds, by the cutting of catastrophic glacial meltwater floods and by the Columbia River system.

The most obvious evidence of bedrock slope failures in the basin is the presence of basalt talus slopes fringing the river canyons and abandoned channels. Such slopes made up of clasts of nearly the same size and are standing at near the angle of repose.

Bedrock failures are most commonly in the form of very large ancient slumps or slump flows. Block glides may be locally important and probably result from failures along interbeds or palagonite zones at flow contacts. Most of these ancient failures occur in areas of regional tilting or are associated with folds. The final triggering, in many cases, appears to have been oversteepening of slopes or removal of toe support.

Along I-82 on the west edge of the province and in a structural basin near Pasco, layers of sediments interfinger with basalt flows. Some of these sediments are compact enough to be considered siltstone or sandstone and are rich in montmorillionite. Slumps and translation failures are common in some places along planes sloping as little as 8 degrees. Most landslides are associated with pre-existing failure surfaces developed by folding and or ancient landslides. In the Spokane and Grande Ronde areas thick sections of sediments make up a major part of the landslide complexes.

5.9.7 Latah Formation

Much of eastern Washington is underlain with thick sequences of basaltic flow rock. These flows spread out over a vast area that now comprises what is commonly known as the Columbia Plateau physiographic province (see **WSDOT GDM Section 5.9.6**). Consisting of extrusive volcanic rocks, they make up the Columbia River Basalt Group **(Griggs, 1959)**. This geologic unit includes several basalt formations, each of which includes several individual flows that are commonly separated from one another by sedimentary lacustrine deposits **(Smith et al., 1989)**. In the Spokane area, these sedimentary rock units are called the Latah Formation.

Most of the sedimentary layers between the basalt flows range from claystone to fine grained sandstone in which very finely laminated siltstone is predominant. The fresh rock ranges in color from various shades of gray to almost white, tan and rust. Much of the finer grained strata contain leaf imprints and other plant debris. Because of its generally poorly indurated state, the Latah rarely outcrops. It erodes rapidly and therefore is usually covered with colluvium or in steeper terrain hidden under the rubble of overlying basaltic rocks.

The main engineering concern for the Latah Formation is its potential for rapid deterioration by softening and eroding when exposed to water and cyclic wetting and drying **(Hosterman, 1969)**. The landslide potential of this geologic unit is also of great engineering concern. While its undisturbed state can often justify relatively high bearing resistance, foundation bearing surfaces need to be protected from precipitation and groundwater. Construction drainage is important and should be planned in advance of excavating. Bearing surface protection measures often include mud slabs or gravel blankets.

In the Spokane area, landslide deposits fringe many of the buttes **(Thorsen, 1989)**. Disoriented blocks of basalt lie in a matrix of disturbed inter flow silts. The Latah Formation typically has low permeability. The basalt above it is often highly fractured, and joints commonly fill with water. Although this source of groundwater may be limited, when it is present, and the excavation extends through the Latah-basalt contact, the Latah will often erode (pipe) back under the basalt causing potential instability. The Latah is also susceptible to surface erosion if left exposed in steep cuts. Shotcrete is often used to provide a protective coating for excavation surfaces. Fiber reinforced shotcrete and soil nailing are frequently used to for temporary excavation shoring.

The Latah formation has been the cause of a number of landslides in northeast Washington and in Idaho. Measured long-term shear strengths have been observed to be in the range of 14 to 17 degrees. It is especially critical to consider the long-term strength of this formation when cutting into this formation or adding load on this formation.

5.9.8 Seattle Clay

The Seattle clay consists of proglacial lacustrine deposits of silt and clay that were deposited during the transition from the interglacial to glacial period. The Seattle clay has been glacially consolidated and is typically very stiff to hard. The Seattle clay is encountered in the Puget Sound Lowland as a discontinuous deposit. It is the primary deposit affecting engineering design in the drumoidial hills located in Seattle along SR-5 and on Mercer Island. This deposit can be more than 50 meters thick in the Seattle area (**Troost and Booth, 2003**).

As a result of the glacial consolidation, the Seattle clay unit generally has high locked in lateral stresses. The locked in stresses have created problems in virgin excavations into this geologic unit. Fractures and slickensides are commonly encountered in this unit. As excavations are completed, the unit experiences a lateral elastic rebound which has led to slope instability and problems for shoring contractors if the locked in stresses and rebound are not incorporated into the design. The failure mechanism consists of joints and fractures opening up upon the elastic rebound response. Hydrostatic pressure buildup within the joints and fractures can then function as a hydraulic jack to further displace blocks of the silt and clay. Appreciable movement can drastically degrade the shear strength along the planes of movement leading to progressive failures. Such instability occurred in the downtown Seattle area when cuts were made into the Seattle Clays to construct Interstate 5.

Based on considerable experience, the long-term design of project geotechnical elements affected by this geologic unit should be based on residual strength parameters. For Seattle clays, the relationship between the residual friction angle and the plasticity index as reported in **NAVFAC DM7** generally works well for estimating the residual shear strength (see **Figure 5-1**). In practice, residual shear strength values that have been estimated based on back-analysis of actual landslides is in the range of 13 to 17 degrees.

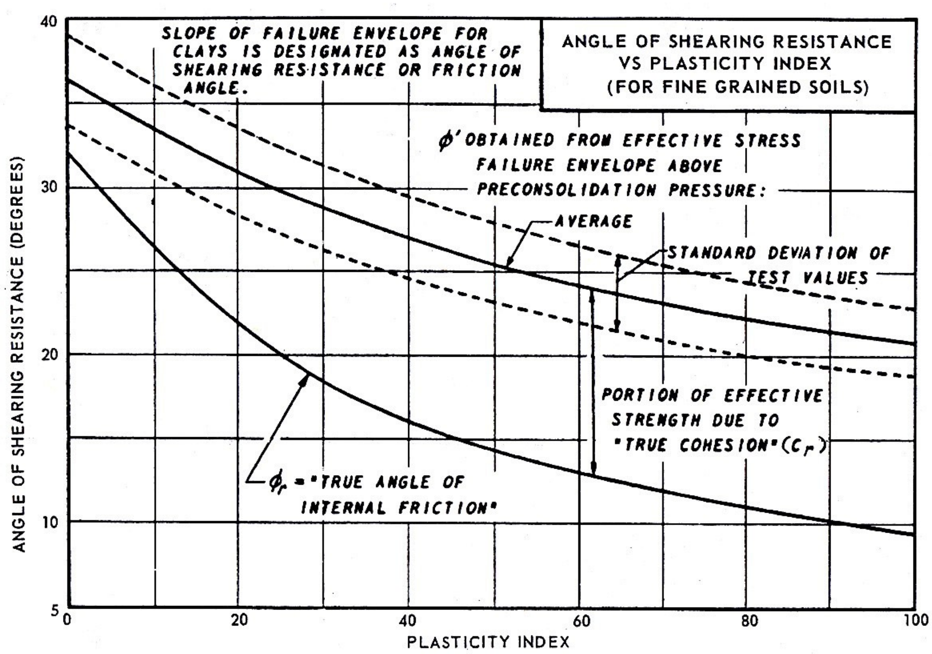

Figure 5-1. Correlation between residual shear strength of overconsolidated clays and plasticity index (**after NAVFAC, 1971**).

It is important to note that residual strength of this formation is often achieved with a shear strain of 5% of less as measured in triaxial or direct shear devices. For plane strain conditions, which typically govern in slope stability and retaining wall applications, this critical shear strain could be significantly less than this.

Another feature of this deposit is that it tends to be laminated with fine, water bearing sands. This situation often results in instability, not only in open cuts, but also in the form of caving in relatively small diameter shaft excavations.

Along the sides of these drumoidial hills containing Seattle clay are deposits from landslides that occurred between glacial periods as the glaciers temporarily receded. Subsequent glacial advances partially reconsolidated these ancient landslide deposits, but in the process left a highly fractured and jumbled deposit of tilted clay blocks and displaced soil. These deposits can be highly unstable, and undrained or peak drained shear strength parameters should not be used even for temporary designs or for wall types that do not allow displacement (i.e., cylinder pile or tieback walls that are designed for K_0 conditions).

As with most fine grained soils, wet weather construction in Seattle clay is generally difficult. When the moisture content of these soils is more than a few percent above the optimum moisture content, they become muddy and unstable and operation of equipment on these soils can become difficult.

Even though this geologic deposit is a clay, due to the highly overconsolidated nature of this deposit, settlement can generally be considered elastic in nature, and settlement, for the most part, occurs as the load is applied. This makes placement of spread footings on this deposit feasible, provided the footing is not placed on a slope that could allow an overall stability failure due to the footing load (see **WSDOT GDM Chapter 8**).

5.9.9 Bellingham Glaciomarine Drift

Glaciaomarine drift typically consists of unsorted, unstratified silt and clay with varying amounts of sand, gravel, cobbles and occasional boulders. Glaciomarine drift is derived from sediment melted out of floating glacial ice that was deposited on the sea floor. This material locally contains shells and wood.

The Bellingham area glaciomarine drift is found west of the Cascade Mountains and from about the Mount Vernon area north. It is typically found at the surface or below Holocene age deposits. The upper portion of this unit, sometimes to about 15 feet of depth, can be quite stiff as a result of desiccation or partial ice contact in upland areas. This material typically grades from medium stiff to very soft with depth. The entire glaciomarine drift profile can be stiff when only a thin section of the drift mantles bedrock at shallow depths. Conversely, the entire profile is typically soft in the Blaine area and can be soft when in low, perennially saturated areas.

This geologic unit can be very deep (150 ft or more). The properties of this unit are extremely variable, varying as a function of location, depth, loading history, saturation and other considerations. The soft to medium stiff glaciomarine drift has very low shear strength, very low permeability and high compressibility. Based on vane shear and laboratory testing of this unit, the soft portion of this unit below the stiff crust typically has undrained shear strengths of approximately 500 to 1000 psf, and can be as low as 200 to 300 psf. The upper stiff crust is typically considerably stronger, and may be capable of supporting lightly loaded footing supported structures. Atterberg limits testing will typically classify the softer material as a low plasticity clay; although, it can range to high plasticity. Consolidation parameters are variable, with the compression index (C_C) in the range of 0.06 to over 0.2. Time rates of consolidation can also be quite variable.

Wet weather construction in glaciomarine drift is very difficult because of the relatively high clay content of these soils. When the moisture content of these soils is more than a few percent above the optimum moisture content, glaciomarine drift soils become muddy and unstable, and operation of equipment on these soils can become very difficult. Localized sandy and gravelly layers in the drift can be saturated and are capable of producing significant amounts of water in cuts. Glacial erratics, such as cobbles and boulders, can also be present.

5.9.10 Coastal Range Siltstone/Claystone

The Coast Range, or Willapa Hills, are situated between the Olympic Mountains to the north and the Columbia River to the south. Thick sequences of Tertiary sedimentary and volcanic rocks are present. The rocks are not intensely deformed and were not subjected to tectonism or the associated metamorphism **(Lasmanis, 1991)**. The Willapa Hills have rounded topography and deep weathering profiles. The interbedded sandstone and fine grained sedimentary formations are encountered in highway cuts. The material from these cuts has been used in embankments. Some of the rock excavated from these cuts will slake when exposed to air and water and cause settlement of the embankment, instability and pavement distortion.

Locally thick clay soils are present and extensive areas are underlain by sedimentary and volcanic rocks that are inherently weak. Tuffaceous siltstone and tilted sedimentary rocks with weak interbeds are common. The volcanic units are generally altered and or mechanically weak as a result of brecciation. The dominant form of landslide in this area is the earthflow or slump-flow **(Thorsen, 1989)**. Many of these are made up of both soil and bedrock. Reactivation of landslide in some areas can be traced to stream cutting along the toe of a slide.

5.9.11 Troutdale Formation

The Troutdale Formation consists of poorly to moderately consolidated silt, sand and gravel deposited by the Columbia River. These deposits can be divided into two general parts; a lower gravel section containing cobbles, and upper section that contains volcanic glass sands. The formation is typically a terrestrial deposit found in the flood plane of the Columbia River and the Portland Basin. The granular components of the formation are typically well-rounded as a result of the depositional environment and are occasionally weakly cemented. Occasional boulders have been found in this formation. Excavation for drilled shafts and soldier piles in these soils can be very difficult because of the boulders and cemented sands.

Slope stability issues have been observed in the Troutdale Formation. Significant landslides have occurred in this unit in the Kelso area. Wet weather construction can be difficult if the soils have significant fines content. As described above, when the moisture content of soil with relatively high fines content rises a few percent above optimum, the soils become muddy and unstable. Permeability in this soil unit varies based on the fines content or presence of lenses or layers of fine grained material.

5.9.12 Marine Basalts - Crescent Formation

The Crescent Formation basalts were erupted close to the North American shoreline in a marine setting during Eocene time **(Lasmanis, 1991)**. The formation consists mostly of thick submarine basalt flows such as pillow lavas. The Crescent Formation was deposited upon continentally derived marine sediments and is locally interbedded with sedimentary rocks. The Crescent Formation extends from the Willapa Hills area to the Olympic Peninsula. During the middle Eocene, the Crescent Formation was deformed due to accretion to North America. The pillow basalts have extensive zones of palagonite. Along the Olympic Peninsula the basalts are generally highly fractured and are often moderately weathered to decomposed.

The properties of the marine basalts are variable and depend on the amount of fracturing, alteration and weathering. Borrow from cut sections is generally suitable for use in embankments; however, it may not be suitable for use as riprap or quarry spalls because of degradation. All marine basalts should be tested for degradation before use as riprap or quarry spalls in permanent applications.

5.9.13 Mélange Rocks on Olympic Peninsula

During the middle Miocene, convergence of the Juan de Fuca plate with the North American plate accelerated to the point that sedimentary, volcanic, and metamorphic rocks along the west flank of the Olympics were broken, jumbled, and chaotically mixed to form a mélange **(Thorsen, 1989)**. This formation is known as the Hoh rock assemblage. Hoh mélange rocks are exposed along 45 miles of the western coast. Successive accretionary packages of sediments within the core of the mountains are composed of folded and faulted Hoh and Ozette mélange rocks. Typical of mélange mixtures, which have been broken, sheared and jumbled together by tectonic collision, the Hoh includes a wide range of rocks. Extensively exposed in headlands and terraces along the Olympic coast consisting of resistant sandstone and conglomerated sequence. The mélange rocks may consist of pillow basalt, deep ocean clay and submarine fans. Slopes in tilted sedimentary rocks that have been extensively altered and/or contain weak interbeds have been undercut by wave action in places along the Strait of Juan de Fuca. Slump flows or bedding plane block glides form along the interbeds.

Because of the variability of the mélange rocks and the potential for failure planes, caution should be used when designing cuts. An adequate exploration program is essential to determine the geometry and properties of the soil and rock layers.

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